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THREE-DIMENSIONAL FE ANALYSIS OF A NAILED SOIL WALL CURVED IN PLAN

I. M. SMITH* AND N. SU

Division of Civil Engineering, School of Engineering, University of Manchester, Oxford Road, Manchester M13 9PL, U.K.

SUMMARY

A nailed soil wall curved in plan was modelled in three-dimensions by the finite element method for construction, service and ultimate loading conditions. The behaviour of the nailed soil wall, the soil–nail interaction, the role of the reinforcement, and the overall and internal failure mechanisms were investigated.

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(No. of Figures: 18 No. of Tables: 1 No. of Refs: 11)

Key words: finite elements; soil nailing

1. INTRODUCTION

Soil nailing as a method of soil reinforcement *in situ* has been increasingly used to stabilize soil in excavations or improve the stability of existing slopes. Soil nailing constructions usually involve three basic processes: excavation, nailing installation and face stabilization. The nails, which are bars with relatively small sections, are inserted into the ground by either drilling and grouting or by driving.

Soil nailing is a relatively new technique to reinforce soil structures. Compared with reinforced earth walls, although with many similarities, soil nailing clearly demonstrates many differences.[1,2] The different construction procedures between these two cases lead to different stress states and deformations. In contrast to the stress increases during the reinforced earth wall building process, soil nailing usually involves stress relief. Due to the features of the reinforcement in soil nailing, the development of the strengthening mechanism in the nailed soil may be very different to that operating in the usual soil-strip reinforced earth wall. There has been debate on the role of nail bending and shear resistance and the corresponding soil bearing capacity in the strengthening mechanism. So far, theoretical argument and field observation do not give any conclusion on the issue. The strengthening and failure mechanisms may be affected by many factors, such as the properties of nails, the arrangement of nails in construction, construction methods, service conditions, etc. The overall failure mode may be changed from a one-block failure mechanism in rotation to a translative two-block failure mechanism when the positions of service loading are moved.[3] As well as the overall failure of the soil structure, nailed soil presents more potential failure modes which may include breakage of the nails, bending overstress with plastic hinges in

*Correspondence to: I. M. Smith, Division of Civil Engineering, School of Engineering, University of Manchester, Oxford Road, Manchester M13 9PL, U.K.

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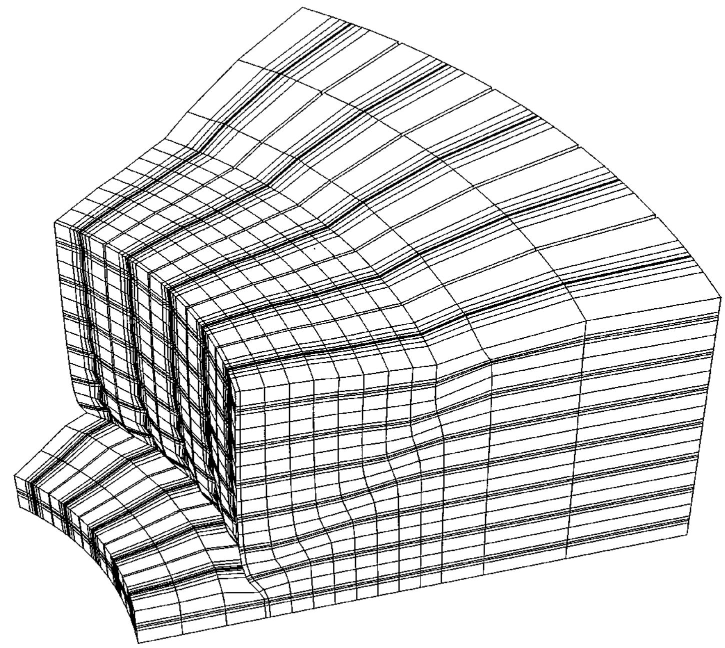

Figure 1. Configuration of the nailed soil wall

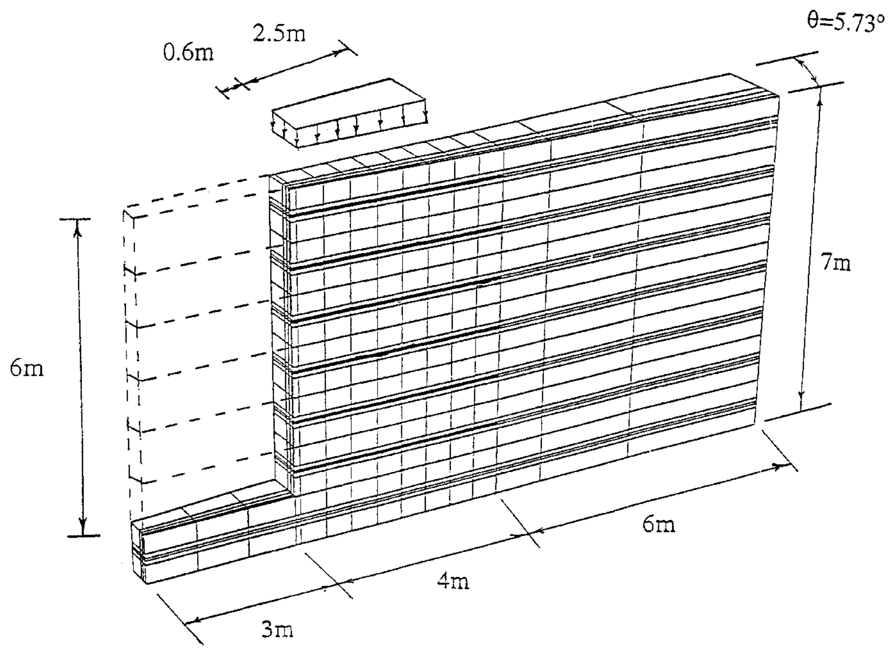

Figure 2. FE mesh and geometry of the slice of the wall

nails, failure of adhesion between the nail and soil and failure of the adjacent soil. Different design methods therefore should be used to ensure both the overall and internal stabilities of the soil structures. Equilibrium should be demonstrated on all of the possible failure mechanisms using assumed properties for the soil and the reinforcement, and assumed loading in the structures.[4]

2. FINITE ELEMENT ANALYSIS

Finite element methods have been successfully used to model reinforced soil constructions. The composite material model or the discrete element model are used in the analysis. In the composite method, the reinforcement and soil are modelled as one material having properties representative of the composite in each element.[5] In the discrete method, the reinforcement and soil are modelled as discrete elements which interact with each other. Detailed information about the interaction between soil and reinforcement can be directly obtained. Therefore the discrete method can be applied to both the external and internal failure states of reinforced soil structures.

Plane strain conditions were assumed in most cases of the previous finite element analyses. This is only valid if the reinforcement is in the form of sheets and wide enough. In soil nailing, due to the geometry of the reinforcements (bars with relatively small section), two-dimensional plane

Table I. Properties of soil, shotcrete and nail assumed in the FE analyses

Properties	Soil	Shotcrete	Nail
E (kPa)	10^5	10^5	2×10^8
ν	0·3	0·3	0·3
c (kPa)	5	10^{10}	10^{10}
ϕ (°)	40	0	0
ψ (°)	0	0	0
γ (kN/m^3)	16	16	16

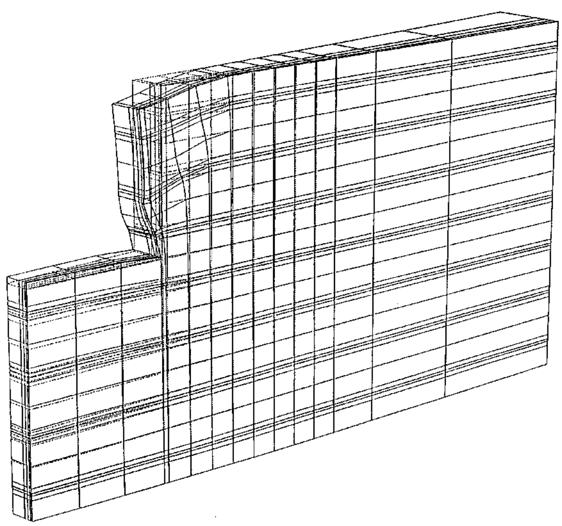

Figure 3. Distribution of displacements of non-nailed wall at collapse

strain finite element analysis cannot appropriately model the stress-transfer in the soil from above a nail to below, and cannot investigate the stress and strain states around the reinforcements and the internal failure mechanisms. Three-dimensional analysis is therefore necessary to model comprehensively the soil nailing behaviour and study the wide range of failure mechanisms.

In the present analysis, an elasto–viscoplastic algorithm, based on the program codes by Smith and Griffiths,[6] was developed to model the various procedures: excavation, nail installation, facing stabilization, under service loading and at collapse. An elastic perfectly plastic stress–strain behaviour was assumed and the material obeyed the Mohr–Coulomb yield criterion. A

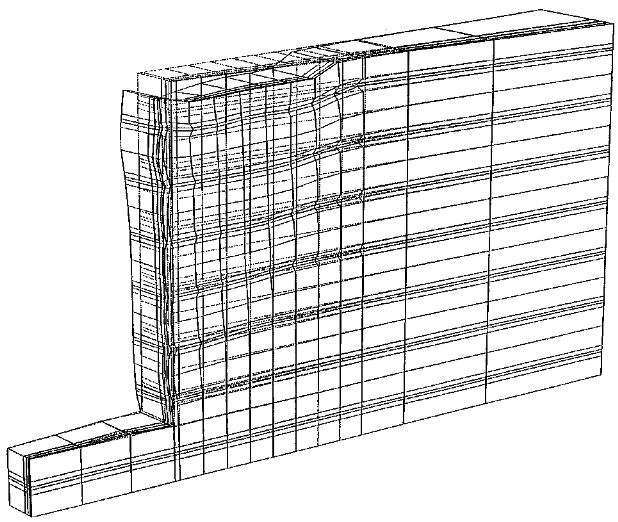

Figure 4. Distribution of displacement vectors of nailed wall at collapse

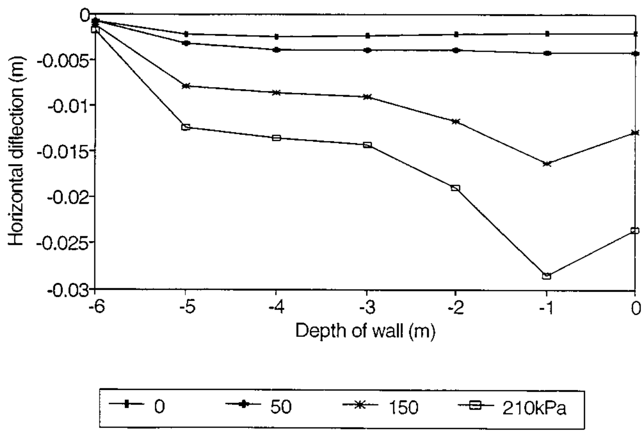

Figure 5. Distribution of horizontal deflections

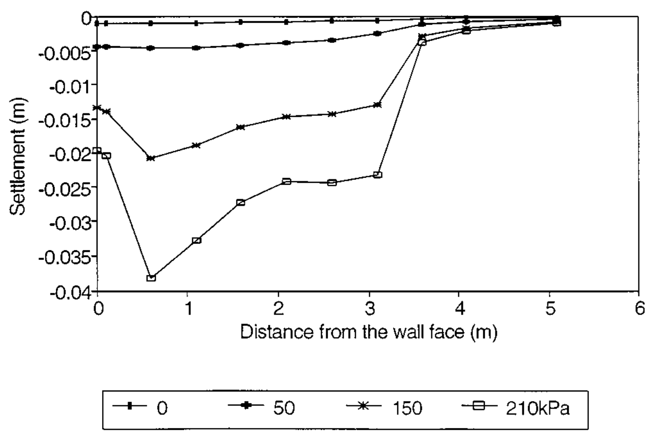

Figure 6. Distribution of settlements

no-tension criterion for soil was also incorporated.[7] Fourteen-node brick elements were employed and $2 \times 2 \times 2$ Gaussian integrating points were used to form the element stiffness. These give more flexible response than 8-node bricks and less computational memory storage than 20-node brick elements.[8] Polar coordinates were used to model the curvature of the wall in the analysis.

Because of the horizontal and vertical repetitive arrangement of the nails, only a slice of the wall between the vertical plane through the nail centrelines and the vertical plane midway between successive columns of nails was analysed. Appropriate boundary conditions were applied to ensure representation of the repetitiveness. This technique was successfully used to model reinforced soil structures.[7,9,10] The configuration of the wall is shown in Figure 1. The FE mesh and geometry of the slice of the wall is shown in Figure 2. The curvature radius to the front boundary was 7 m, to the excavation face 10 m and to back of the fill 20 m. The slice width of the excavation face was 1 m. The nail length was 4 m with cross-section 50×50 mm. The shotcrete thickness was 100 mm.

The initial horizontal stresses in the soil were set up according to a K value of 1. No water was included in the analysis. The construction processes were modelled by the excavation of 1 m of soil followed by the placement of nails and shotcrete. The nail and shortcrete elements were modelled by switching the corresponding soil element's properties into those of nail or shotcrete. The soil disturbance due to the nail installation was ignored. The nails were modelled by the 14-node brick elements without any special interface elements, but surrounded by thin soil elements. A full frictional interaction was therefore assumed. The process of soil excavation and nail/shotcrete placement continued until the required excavation depth (6 m) was reached. The strip surcharge load was then applied stepwise until the displacement increments failed to converge to the given tolerance in the displacement increment norm within the given iteration limit. The load at this stage is referred to as the 'collapse load'. The strip load was 2·5 m broad and situated in a distance of 0·6 m from the excavation face.

The material properties used in the analysis are shown in Table I.

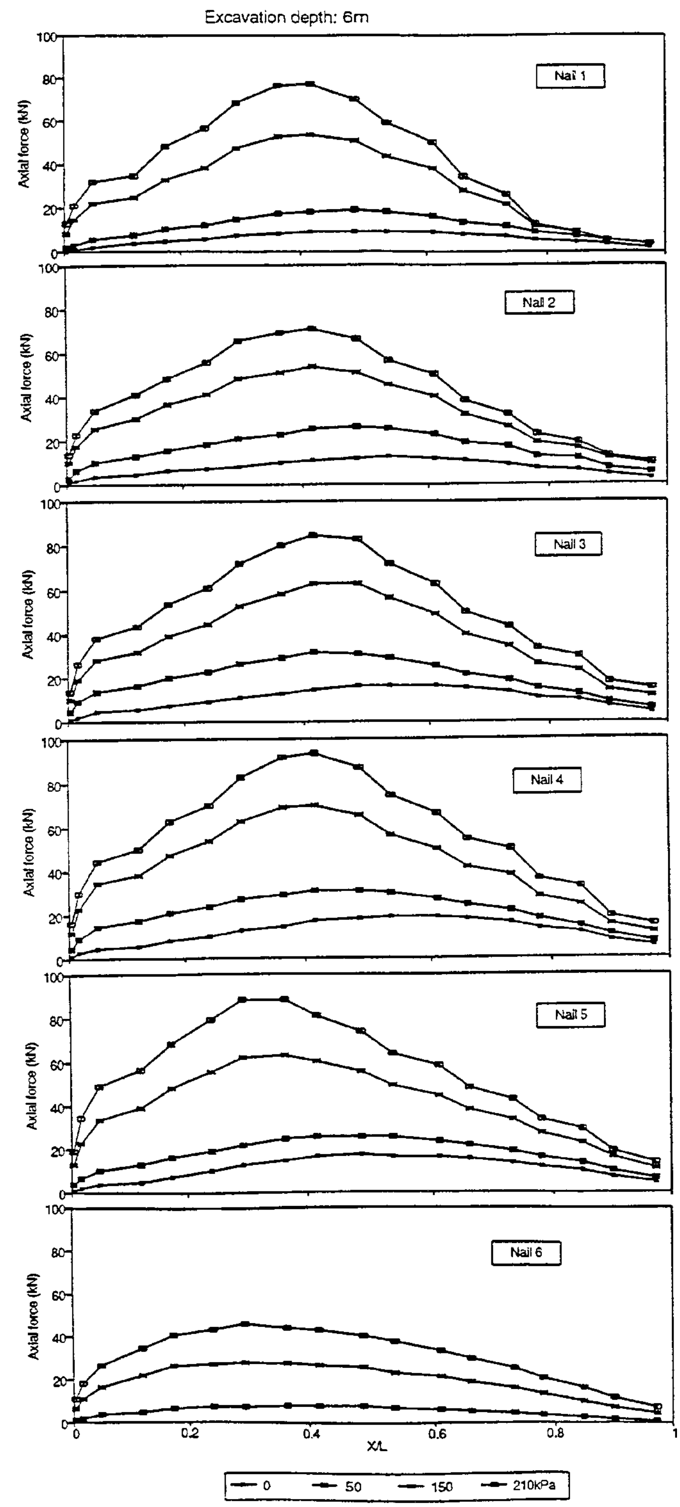

Figure 7. Distribution of axial forces (kN) in nails

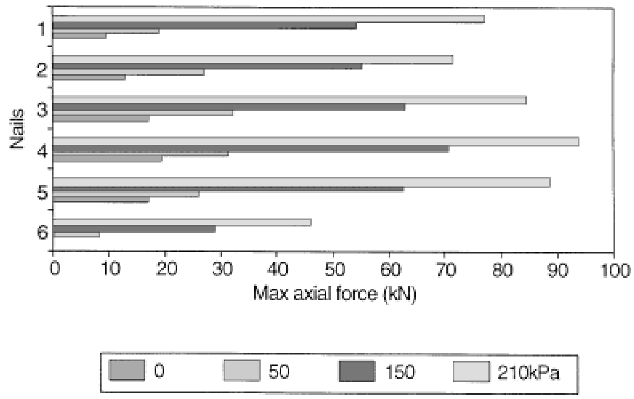

Figure 8. Maximum axial forces in nails

3. RESULTS AND DISCUSSIONS

At the beginning of the analyses, the effect of using various meshes was analysed. In the end the slice of wall was modeled by $5 \times 15 \times 35$ elements. The preliminary results showed that finer meshes gave essentially similar results.

The results during construction (excavation depth to 6 m, no surcharge), under service loads (surcharges of $50\,kN/m^2$, $150\,kN/m^2$) and at ‘collapse’ (surcharge of $210\,kN/m^2$) are presented. Due to the construction processes, the 6th nail was installed after the excavation (6 m) was completed, but before applying the surcharge. Therefore the 6th nail was absent for the result of excavation at depth 6 m and no surcharge. The significant effects of soil nailing on the stability of the wall during excavation can be seen from Figures 3 and 4. Compared with a nailed soil wall which was stable at excavation depth $=6$ m, the non-nailed wall collapsed when the excavation reached a depth of 3 m. The sharp increase of settlement under critical loading, as usually happens in a non-reinforced wall, did not appear under the given tolerance and iteration limit.

3.1. Horizontal deflections and settlements

The horizontal deflections of the wall and surface settlements are shown in Figures 5 and 6. A large horizontal deflection and vertical settlement were caused by the surcharge at ‘collapse’. The horizontal deflection at the top half of the wall increased more rapidly when the surcharge was applied and the largest horizontal deflection appeared at the position of 1 m depth at the wall under a load $210\,kN/m^2$. The increased settlement under surcharge was concentrated in the loading area and the largest settlement appeared at the front of surcharge position at ‘collapse’. However, the position of the largest horizontal deflection and vertical settlement is sensitive to the position of the strip load and was not quite the same as measured in field trials.[3,11]

3.2. Axial forces in nails

The axial force distributions in the nails are presented in Figure 7. Corresponding to the surcharge loading process axial tension forces were mobilized in all nails. The positions of the maximum tension in the nails moved close to the slip line of the structure. The maximum axial

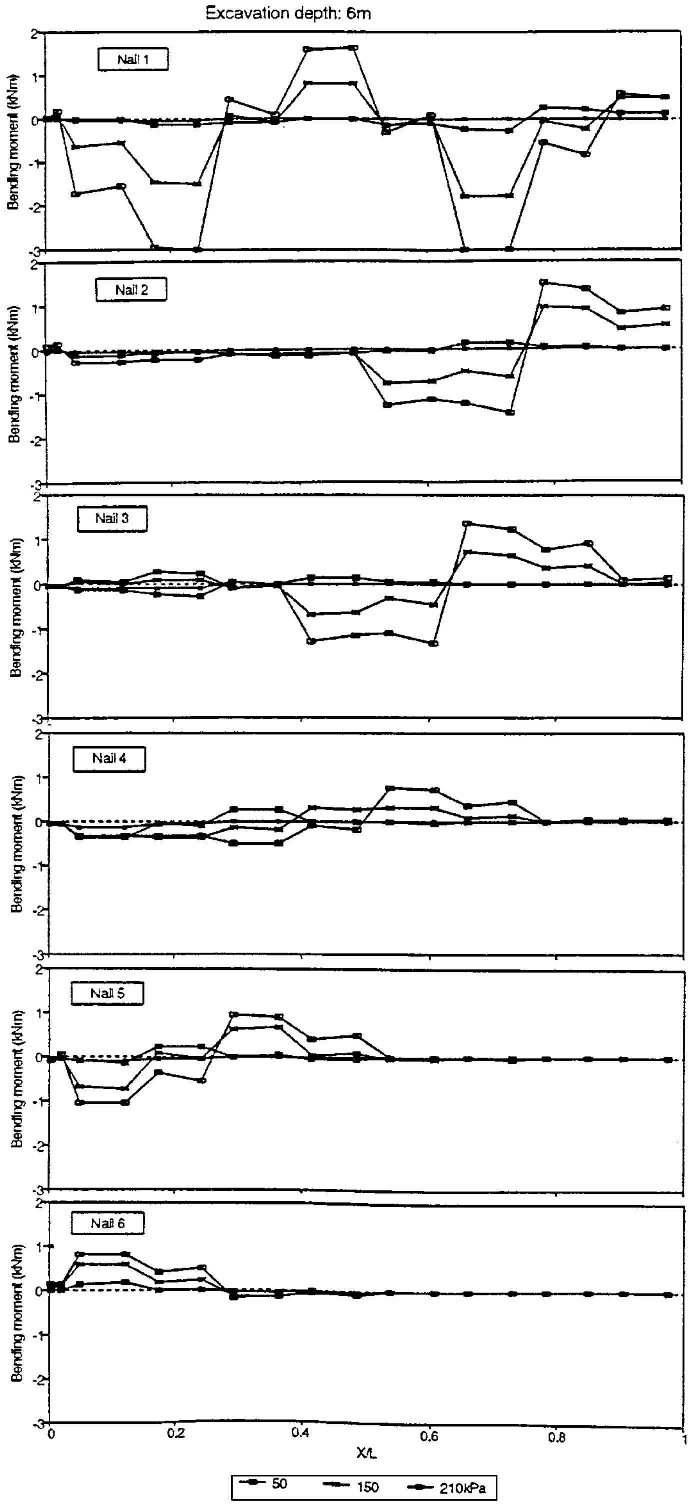

Figure 9. Distribution of bending moment (kN m) in nails

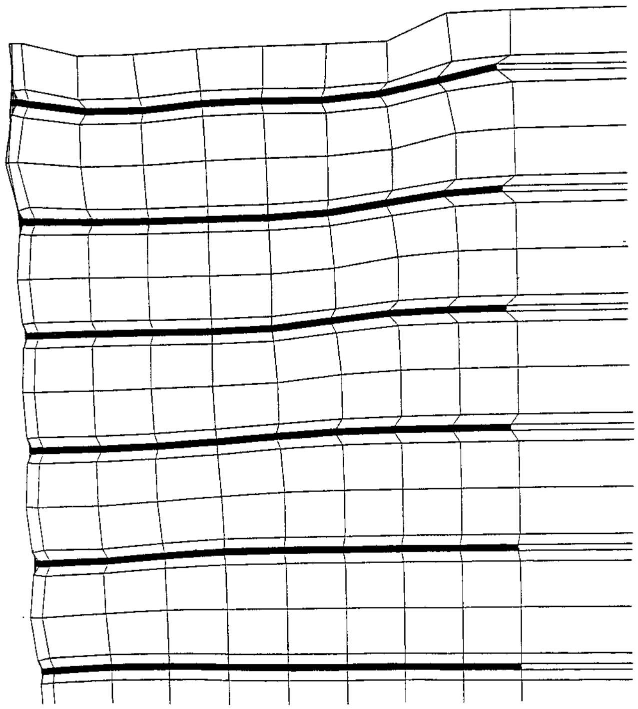

Figure 10. Deformation of nails

forces in the nails are shown in Figure 8. The maximum tension force occurred at the 4th nail. But it might appear at other nails or even change from one nail to another during the loading process.[11]

3.3. *Bending moments in nails*

The distributions of the bending moments in the nails are given in Figure 9. Bending moments are plotted on the tension side of the nail. Compared with the axial force, the bending moments were very small after the excavation was just completed (surcharge $=0$), but mobilized when the surcharge was applied. In contrast to the axial forces, the largest bending moments were observed in the top nails which were near the surcharge area. The deformations of the nails are shown in Figure 10 and we can observe that the bending moments were consistent with the way the nails bend.

3.4. *Shear forces in nails*

The distributions of shear forces in the nails are presented in Figure 11. Compared to the axial forces, bending moments did not generate significant shear forces perpendicular to the nail under small surcharge. However, significant shear forces were generated under large surcharge and the largest shear force occurred in the top nail.

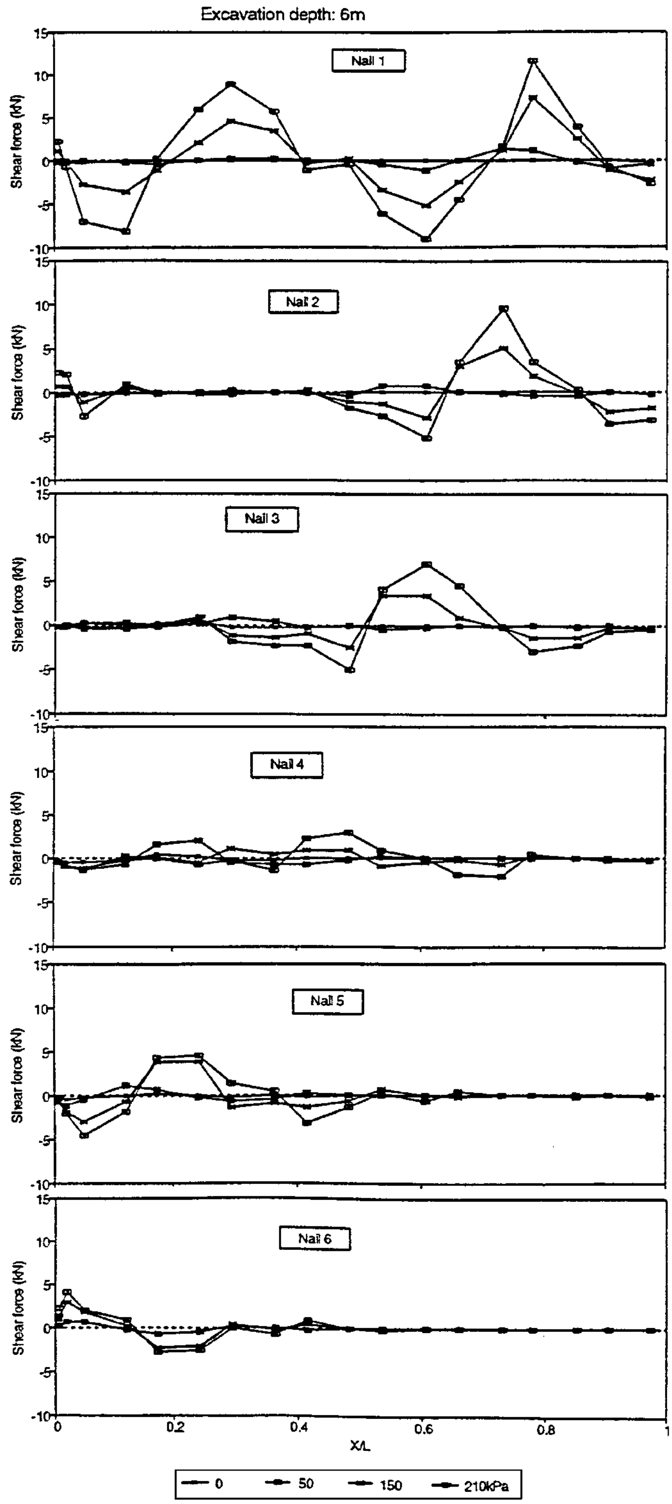

Figure 11. Distribution of shear forces in nails

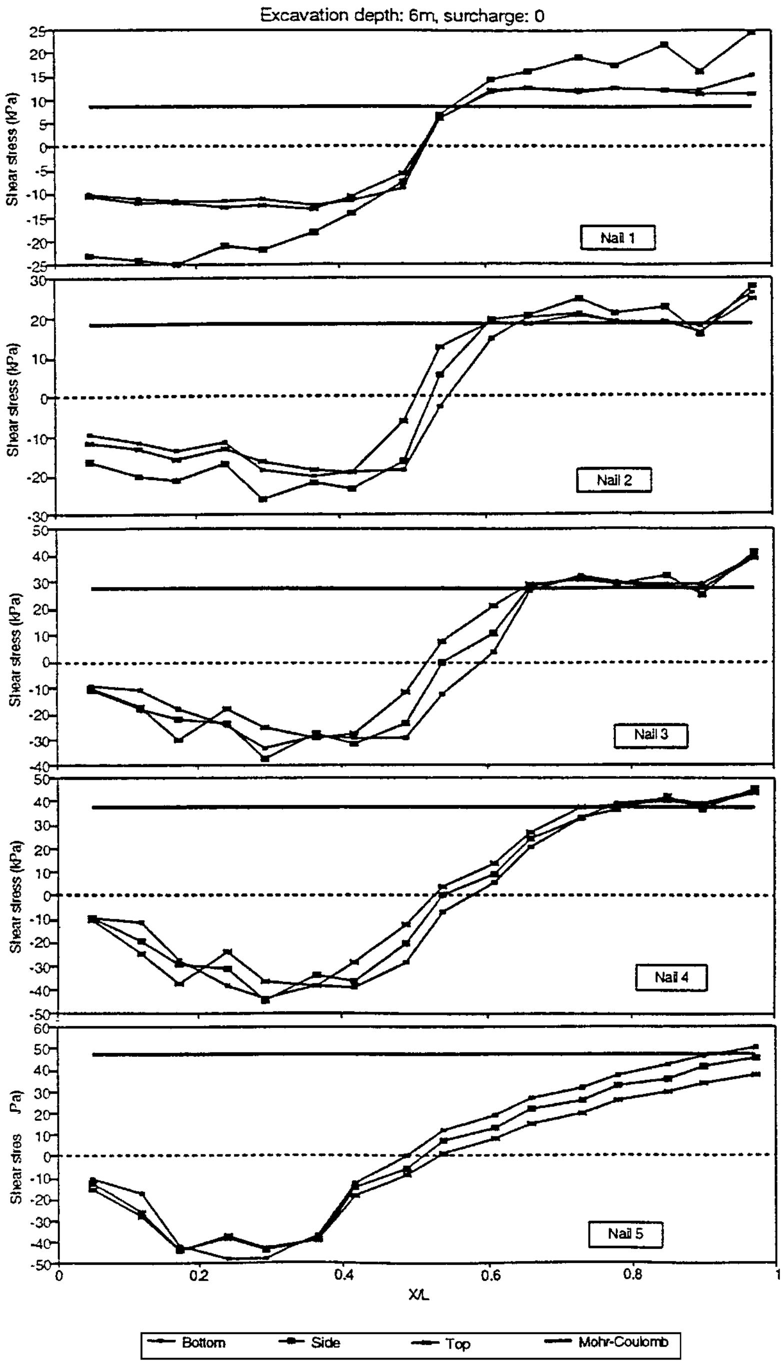

Figure 12. Distribution of shear stresses at the interface of nail/soil

There has been much debate on the role of nail bending and shear resistance and the corresponding soil bearing capacity in the strengthening mechanism. The results indicate that the role of nail bending and shear resistance are different during construction processes, under service loading and at collapse.

3.5. *Shear stresses at soil/nail interfaces*

The distributions of shear stresses at soil/nail interfaces are shown in Figure 12. The shear stresses at the interfaces were obtained by extrapolating the stresses at the Guass points. The shear stresses preventing pullout of nails are given positive signs in the figure. The distribution of the shear stresses clearly demonstrates that the part of nails inserted into the stable block of soil prevented pullout failure, and the other part resisted the slip of unstable soil.

If shear stresses $\tau_{r\theta}$ and $\tau_{\theta z}$ are ignored and assuming $K=1$, the Mohr–Coulomb criterion can be expressed as follows:

$$\tau_{rz} = c\cos\phi - \sigma_z \sin\phi$$

According to this Mohr–Coulomb criterion, at the right end parts of the top nails, the shear resistance of soil at the interfaces was fully mobilized even without surcharge (Figure 12).

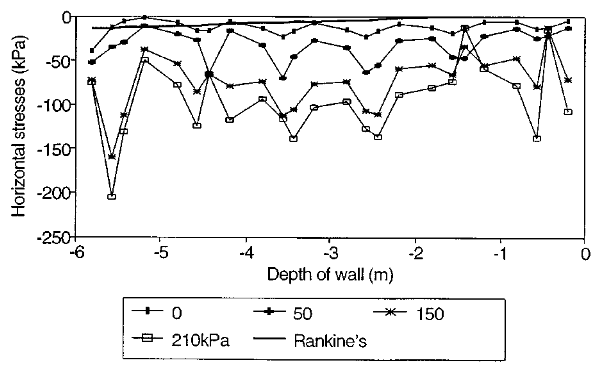

Figure 13. Distribution of horizontal stresses behind the face through nails

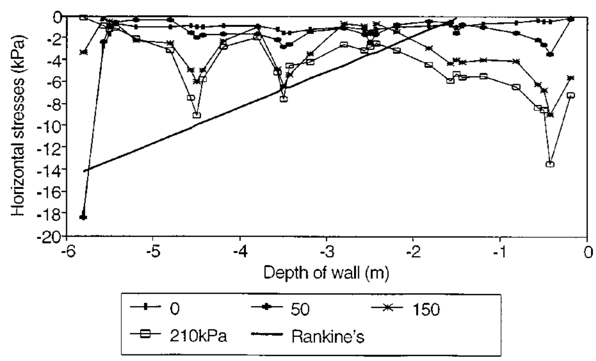

Figure 14. Distribution of horizontal stresses behind the face between nails

3.6. *Horizontal stresses behind the wall face*

The horizontal stress distributions behind the facing through nails and between nails are shown in Figures 13 and 14. The largest values occurred in the vicinity of depth 4.5 m. The distributions and magnitudes of the horizontal stresses through the nails were quite different to those between nails. The horizontal stresses between nails were increased more significantly at the top part of the wall after loading.

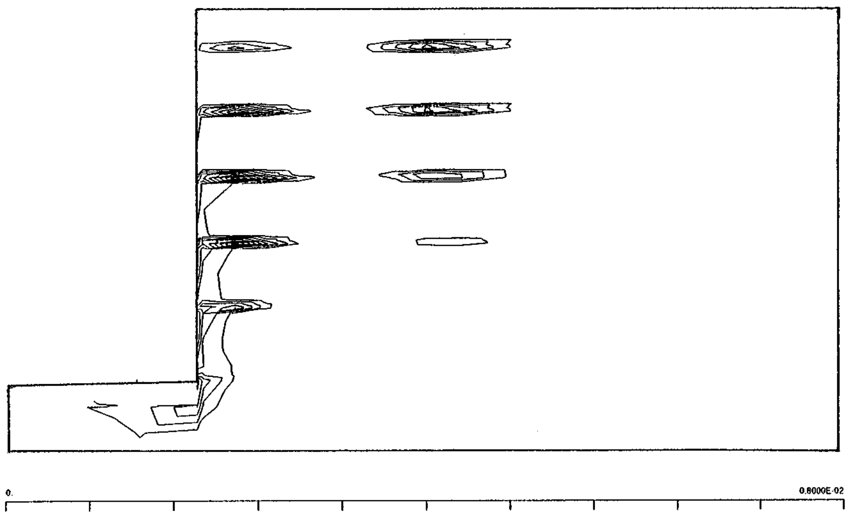

Figure 15. Plastic shear strain distribution through nails (depth = 6 m, surcharge = 0), contour interval: 0·0008

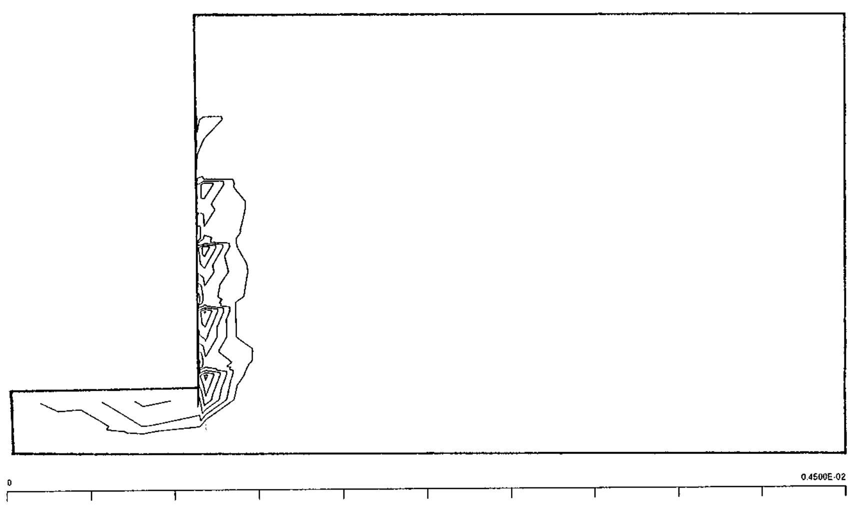

Figure 16. Plastic shear strain distribution between nails (depth = 6 m, surcharge = 0), contour interval: 0·00045

3.7. *Plastic shear strain distributions*

The plastic shear strain distributions through the nails and between nails at the end of construction (surcharge =0) and 'collapse' load (surcharge =210 kPa) are presented in Figures 15–18. At the end of construction, although high plastic shear strain appeared at the bottom half of the

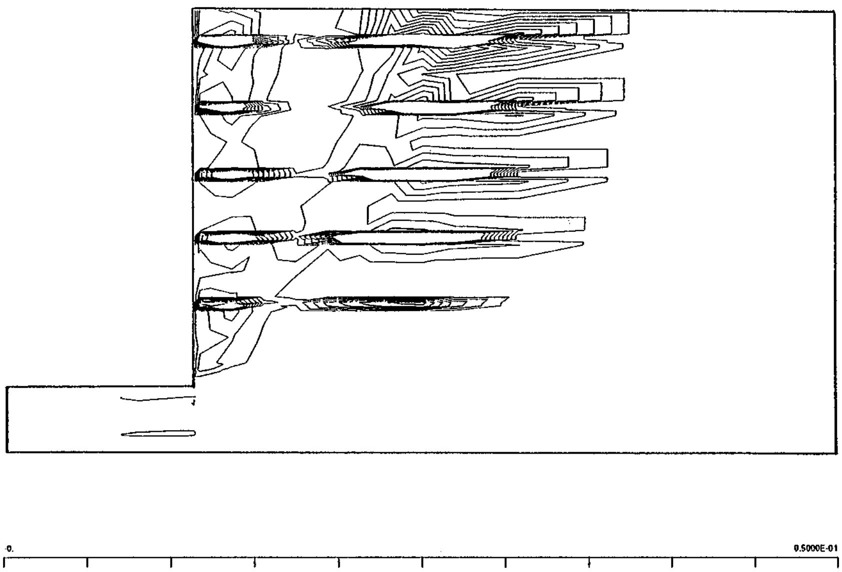

Figure 17. Plastic shear strain distribution through nails (depth =6 m, surcharge =210 kPa), contour interval: 0·005

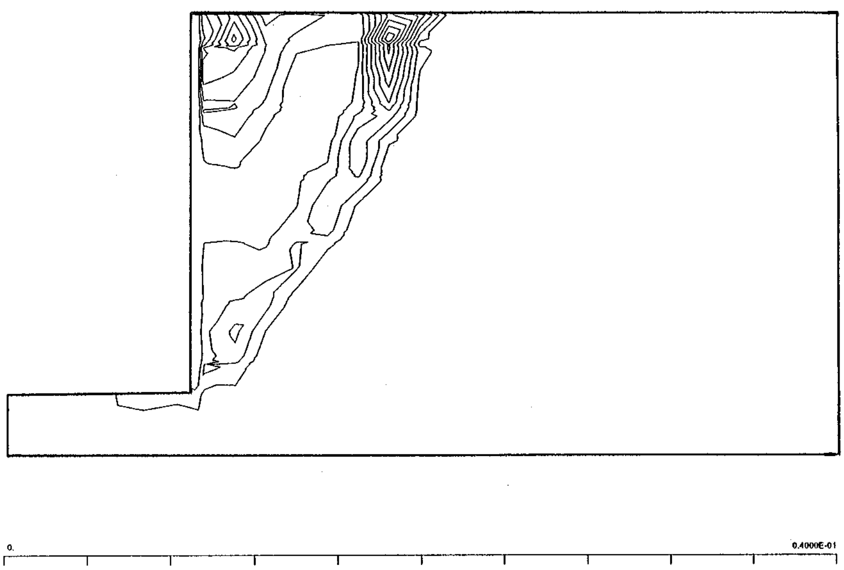

Figure 18. Plastic shear strain distribution between nails (depth =6 m, surcharge =210 kPa), contour interval: 0·004

wall surface area, a shear band was not formed and the overall soil was held by the nails. A shear band was observed at 'collapse' and intersected nails 1, 2, 3, 4 and 5. A high plastic shear strain concentration also appeared in the front local area of the surcharge position. This indicated that the nailed wall failed as a one-block mechanism in conjunction with a local failure. However, a pullout failure mechanism could still happen as the high concentration of contours occurred in the right parts of nails, which were located in the stable block of the soil. To prevent pullout failure happening, the nails should be long enough to intersect the stable block of soil and then provide strong enough pullout resistance. On the other hand, if the strength of the nails is lower than the pullout resistance, increasing axial tension force may induce a nail breakage mechanism. The failure pattern and position of the slip line may change as the position of the surcharge loading alters and this may induce different failure mechanisms.[3,11] The result of Su and Smith[11] indicated a clear one-block failure mechanism without local failure happening.

Therefore in soil nailing structure design, every possible failure mechanism should be investigated and equilibrium should be demonstrated in all possible failure mechanisms in service conditions.

4. CONCLUSIONS

A three-dimensional finite element analysis can model soil nailing construction, under service loading and at collapse successfully. The soil–nail interaction can be investigated and therefore overall and internal failure mechanisms can be analysed. The information obtained from the three-dimensional FEM analysis contributes to the deeper understanding of the behaviour of the reinforcing and failure mechanisms of nailed soil and can be helpful for the correct design of nailed soil structures. Soil nailing technology significantly improves the stability of a wall during excavation and under service loading. The failure mechanisms may change for different loading conditions. The role of bending moment and shear resistance in the nails may be different during construction, under service loading and at collapse.

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